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SYNOPSIS Cyclic drained simple shear tests on a dry sand using a 12 in diameter sample with sample heights of 1, 2, and 4 in show the affect of Diameter/Height ratio on the shear modulus and percent of critical hysteretic damping at various shear strain levels. The shear modulus is found to increase with cycle number and with increasing specimen size. The D/H ratio is found to affect the shear modulus at low shear strains (< 1 percent) and found to have little effect at higher shear strains and at failure. The hysteretic damping decreases for all values of shear strain tested (0.01 to 1 percent) as the cycle number and D/H ratio increases. Possible implications on design and pore pressure development are mentioned.

INTRODUCTION

Much research has been accomplished in the past few years on the study of dynamic soil properties using various laboratory and field measurement techniques. The object of many of these studies is to evaluate the cyclic stress-strain properties and liquefaction potential of soil deposits under earthquake loading conditions.

Currently, dynamic soil properties are being evaluated using resonant column tests (solid and hollow cylindrical specimens), cyclic triaxial compression tests (solid cylindrical specimens), cyclic simple shear tests (cylindrical and square specimens), torsional simple shear tests (solid, hollow, and hollow with variable cross-section specimens), and shaking table tests. The advantages and disadvantages of each of these devices and those of insitu tests have been summarized by Woods (1978), while Krizek and Borden (1977) have focused their attention on laboratory tests measuring the stress-strain properties of saturated sands only.

The cyclic triaxial compression test is used extensively and the cyclic simple shear test is used modestly today (1981) for evaluating dynamic soil properties. As discussed by many investigators (Peacock and Seed, 1968; Seed and Peacock, 1971; Finn et al., 1971; and Seed, 1979), the cyclic simple shear test comes closest to approximating the field conditions during dynamic loadings at a reasonable cost in terms of equipment and sample preparation. The use of shaking tables under uni- and multi-directional loading (De Alba et al., 1976 and Pyke et al., 1975) is also an excellent way to simulate field conditions but the cost involved becomes quite high.

Kjellman (1951) developed the first cylindrical simple shear device with a wire reinforced membrane enclosing the sample. Roscoe (1953) pointed out that Kjellman's device cannot have uniform shear stresses across a horizontal circular cross section because the stresses must be tangential to the circular boundary unless the vertical walls of the device transmit vertical shear stresses to the soil sample. On the other hand, the simple shear device described by Roscoe has severe sample preparation problems not readily solved for production testing. For example, problems arise in the preparation of uniformly dense sand specimens, most pronounced at the corners of the sample.

Presently, most cyclic simple shear devices are of the Norwegian Geotechnical Institute (NGI) type, Cambridge type apparatus, or some form of torsional cyclic simple shear device. The NGI device is an improved version of the original device used by Bjerrum and Landva (1966) in their studies of simple shear in clays. The NGI device uses a disc-shaped specimen up to 2 cm high and 8 cm in diameter, which is surrounded by a wire reinforced membrane fitted with a cap and base. The Cambridge device is an improved version of the simple shear box developed by Roscoe (1953). The Roscoe type device has been used by many investigators (for example, Peacock and Seed, 1968; Finn et al. 1971; and Seed and Peacock, 1971) in their studies on liquefaction potential. The torsional cyclic shear device uses disc-shaped specimens either solid or hollow with constant or variable specimen heights (for example, Hvorslev and Kaufman, 1952; Ishihara and Yasuda, 1975; Lade, 1975; Ladd and Silver, 1975; Yoshimi and Oh-Oka, 1973).

Many investigators have looked into the stress conditions imposed on the soil specimen from these various devices to determine what effect these stress conditions have on the measured dynamic properties (Lucks et al., 1972; Prevost and Hoeg, 1976, Pyke, 1978a,b; Shen et al., 1978a,b; Ladd and Silver, 1975; and Wright et al., 1978). In general, the results of these studies show that local stress conditions can greatly affect the measured dynamic properties from these tests. As Seed (1979) points out, to overcome these local stress concentrations, the test specimens must be of sufficient size to overcome these effects.

Very little experimental research has been accomplished on the effects of sample size on the results of simple shear tests. Pyke (1978a) reported on two studies where the effects of varying the height to diameter ratio on test results were studied "but their results appear to have been influenced by other factors so that unequivocal results were not obtained."

More recently, Carroll (1979) performed tests on an NGI device modified for cyclic loading testing two undisturbed clays. The consolidated constant volume tests were performed on samples of two different diameters various sample heights ranging from 0.85 cm to 1.89 cm, after consolidation. The corresponding D/H ratios based on average height values ranged from 3.0 to 9.4. For the static tests, Carroll found the smaller diameter sample was more resistant to shear by about 10 to 15 percent than the larger sample. In performing cyclic tests, he found the smaller diameter samples to be about twice as resistant to shear strain per cycle as the larger diameter samples. Also, the K_0 values obtained from measured lateral strains were found to be directly proportional to sample height and inversely proportional to sample diameter. "No evidence was obtained to suggest that variations in sample height will effect cyclic shear resistance" (Carroll, 1979). Carroll, however, was not concerned with results at much lower shear strains.

On the theoretical side, Shen et al. (1978a,b) have shown that a more uniform shear strain distribution occurs within a given size (diameter) sample as the D/H ratio increases. They explain that for the same amount of horizontal displacment imposed on the simple shear specimen, a smaller external moment develops within the thinner specimen (larger D/H ratio), producing a more uniform shear state. However, at any shear strain, the amount of horizontal displacement required for a 4 in high sample, for example, with a 12 in diameter (D/H = 3) is four times as large as for a 1 in high sample (D/H = 12). The problem of induced moments is exacerbated when comparing test results at the same shear strain.

Franke et al. (1979) reiterate Roscoe's point that a homogeneous state of stress that occurs under field conditions cannot be achieved because complementary shear stresses at the vertical boundaries of the sample specimen cannot be applied. They point out that only Arthur et al. (1977) has been able to achieve the transmission of complementary shear stresses to the specimen.

To reduce the effect of not having complementary shear stresses at the boundary of the sample, it is necessary to increase the diameter to height ratio. To date, adequate D/H ratio has only been achieved in the shaking table tests by De Alba et al. (1976). Kovacs (1973) previously suggested a minimum D/H ratio of 6 be used.

Franke et al. (1979) investigated the effects of D/H ratio using the BAW device while testing saturated sand in the undrained state. They found similar test results (at failure) for ratio of D/H = 7.5 and 3.75. The experimental evidence suggests that at high strains (at failure), the D/H ratio is not an influencial factor; only at low shear strains the D/H ratio is important, based on this study.

Table 1 presents a partial summary of both experimental and theoretical studies conducted using the simple shear device under both static and dynamic loadings. The list is far from complete and does not cover many studies performed outside the U.S., especially in the U.K. and Norway. For the most part, with some exceptions, the studies were performed on low D/H ratios ($\leq$ 4).

To better establish experimentally the effects of the D/H ratio on the shear modulus and hysteretic damping at low strains ($\leq$ 1 percent), cyclic loading tests were performed on 12 in diameter samples with sample heights of 1, 2, and 4 in under equivalent and similar conditions of relative density, normal stress and testing frequency.

TEST APPARATUS AND SAND TESTED

A schematic diagram of the 12 in diameter cyclic simple shear device is presented in Figure 1.

The cyclic simple shear device consists of a top and bottom circular grooved plate (grooves are perpendicular to the direction of loading to facilitate plate roughness) while a series of stacked rings outside a rubber membrane encloses the specimen. A pneumatic loading piston provides a vertical or normal load to the sample while a servo-hydraulic actuator provides the cyclic horizontal load. The top of the 12 in diameter sample is held "rigid" by four strain gage type load cells while the bottom of the sample is moved on a track-mounted plate attached to the actuator. All Ottawa 20-30 sand samples were tested at a relative density of 60 percent under a normal pressure of 500 psf (24 kPa) and were subjected up to 300 cycles of a 0.5 Hz sinsoidal strain controlled loading with cyclic shear strains varying from about 0.01 to 1.0 percent. The specimen heights were 1, 2 and 4 in (D/H ratios of 12, 6, and 3, respectively). Appropriate instrumentation monitored horizontal load, vertical load and horizontal displacement. An X-Y plotter recorded shear stress and shear strain permitting the evaluation of the (chord or secant) shear modulus and percent of critical hysteretic damping from the hysteresis loops as described by Hudson (1965).

The dry sand samples were formed in the bottom circular plate with the membrane rolled up and around the stacked rings and hand compacted in three layers per inch of sample height using a 4 in tamping foot until the required density was achieved. When the required sample height is attained, the top plate is positioned on the sample and the membrane rolled up and sealed with an O ring. It is believed that the hand tamping pressure did not exceed the test normal pressure.

EXPERIMENTAL RESULTS

The cyclic test results of the shear modulus and of the hysteretic damping-shear strain relationships for the three diameter to height ratios are presented in the following figures. Prévost (1978) has shown theoretically that the simple shear device measures lower shear stress for a given shear strain (and hence lower shear modulus) than an ideal simple shear device which provides shear stresses upon every side of the soil specimen. The shear moduli obtained from this study appear to be lower and the damping higher than those reported in the literature (e.g. Hardin and Drnevich, 1972) and are not intended for use in design. Rather, it is the observed trends that are considered important herein.

Figure 2 ilustrates the normalized shear modulus (shear modulus/confining pressure)-shear strain relationship for three D/H ratios for the 12 in diameter sample. As expected, the shear modulus decreases with increasing shear strain and increases with the number of cycles at a given shear strain. At high shear strain (~ 1 percent), the effect of the D/H ratio is not important, it is only at low shear strains where wide differences of shear modulus appear, as a result of the D/H ratio. At low shear strains, the sand appears stiffer with thick samples (D/H = 3) as compared to thinner samples with the same diameter (D/H = 6, 12). When the test results are replotted in Figure 3 in terms of D/H ratio, the effects of the D/H ratio are apparent at low strains (0.05 percent).

The behavior shown in Figures 2 and 3 for granular materials is also similar to results obtained in testing clays under similar cyclic conditions (Kovacs, 1973). Figure 4 not only shows the effects of the normalized shear modulus-shear strain relationships (G = shear modulus and S_u = undrained shear strength) but shows the influence of sample size as well for L/H = 2. Smaller samples give a larger modulus at low shear strains than larger samples. For clays, the effects of sample size are apparent when the normalized shear modulus versus length to height ratio is plotted in Figure 5 for different sample sizes of unconfined blocks of clay.

Table 1. Partial Summary of Cyclic and Static Simple Shear Studies

Simple Shear Type	Sample Height	Sample Diameter or Size	D/H	Soils Tested	Reference
Rectangular	2 cm	6 x 6 cm	3		Roscoe, 1953
Roscoe	2 cm	6 x 6 cm	3	Sands	Arthur, James and Roscoe, 1964
Roscoe	2 cm	6 x 6 cm	3	Sand	Roscoe, Bassett, and Cole, 1967
Roscoe Type	2 cm	6 x 6 cm	3	Monterey No. 0 Sand @ 1/6, 1, 2 and 4 Hz	Peacock and Seed 1968
Roscoe	2 cm	6 x 6 cm	3	San Francisco Bay mud	Duncan and Dunlop, 1969
NGI	2 cm	8 cm	4	San Francisco Bay mud	Thiers and Seed, 1969
Roscoe Type	1 1/8 in	2 x 2 in	1.6	C109 Ottawa Sand at 2 Hz	Finn, Pickering, and Bransby, 1971
Unconfined Blocks	6 in	12 x 12 in	2	Kaolinite/Bentonite 3:1 mix @ 1, 2, 5 and 10 Hz	Kovacs, Seed and Chan, 1971
NGI	2 cm	8 cm	4	No. 20 Crystal Silica Sand @ 1 Hz	Silver and Seed, 1971
NGI	3 cm	5.4 cm	1.8	Theoretical Studies	Lucks et al. 1972
Unconfined Blocks	2 in 4 in 8 in	2 x 2 in 4 x 4 in 8 x 8 in	1	Kaolinite/Bentonite 3:1 mix @ 1, 2, and 5 Hz	Kovacs, 1973
	1 in 2 in 4 in	2 x 2 in 4 x 4 in 8 x 8 in	2		
	1 in 2 in	4 x 4 in 8 x 8 in	4		
	1 in	8 x 8 in	8		
Torsional Circular	Variable 0.5 in inside to 1.0 in outside	4 in OD 2 in ID	?	C109 Ottawa Sand @ 2 Hz	Ishibashi and Sherif, 1974
NGI	2 cm	8 cm	4	No. 20 Crystal Silica Sand at 0.5 Hz	Park and Silver, 1975
Shaking Table	4 in	80 x 42 in (L x W)	20	No. 0 Monterey Sand, to 4 Hz	De Alba, Seed, and Chan, 1976
NGI Type	3 cm	10.8 cm	3.6	Various	Hara and Kiyota, 1977
NGI	2 cm	8 cm	4	Theoretical FEM studies	Shen, Sadigh, and Herrmann, 1978 Shen, Herrmann, and Sadigh, 1978
Circular Stacked Rings	1 in	2.4 to 4 in	2.4 to 4	None	Sidey, Strom, and Pyke, 1978
NGI	0.85 cm 1.56 cm 0.89 cm 1.6 cm	8 cm 4.76 cm	9.4 5.2 5.4 3.0	Gulf of Mexico clay - Gulf of Alaska clay, 0.1 Hz	Carroll, 1979
BAW	1 to 2 cm	7.5 cm	3.75 to 7.5	Sand	Franke, Kiekbusch, and Schuppener, 1979
NGI	1.3 to 2.0 cm	4.76 cm	2.4 to 3.7	Concord Blue Clay, 0.5 Hz	Kopal, 1979
NGI	0.75	1.88 in 3.15 in	2.5 4.2	Sands to Clays (seven soils)	Weaver and Roth, 1979
Roscoe	2 cm	8 cm	4	Theoretical studies	Wood, Drescher, and Budhu, 1979
NGI	1.7 cm avg.	8 cm	4	Marine Clay, Concord Blue Clay	Zimmie and Floess, 1979
NGI, modified	2 cm	8 cm	4	Icy Bay Marine clay - 1 and 0.05 Hz	Idriss, et al., 1980
Variable,* Confined	2 cm	7 cm	3.5	Monterey No. 0 Sand 0.5 Hz	Silver et al. 1980
Circular	25 cm used (5 to 50 cm available)	70 cm	2.8	Dry Sandy Gravel (GW)	Tokue, Hayashi, and Kitahara, 1980
NGI	2 cm	8 cm	4	Gulf of Mexico Clay - Gulf of Alaska Clay	Dyvik, Zimme and Floess, 1981
NGI	2 cm	8 cm	4	Pacific Illite, Gulf of Mexico Clay	Dyvik, Zimme and Schimelfenyg, 1981
Circular Stacked Rings	1 in 2 in 4 in	12 in	12 6 3	Dry Ottawa No. 20-30 sand at 0.5 Hz	This study

* Can use either: circular with membrane, wire reinforced membrane, or stacked rings; rectangular with membrane or stacked rings. Thinner samples possible.

The D/H ratio also influences the hysteretic damping-shear strain relationship as shown in Figure 6. As expected, the percent of critical hysteretic damping increases with increasing shear strain and decreases with cycle number at a given shear strain. Although the data are scattered, it appears that the damping values decrease with increasing D/H ratios. Figure 7 illustrates this variation of damping in terms of D/H ratio at two shear strains. As the D/H ratio increases, damping decreases.

COMPARISON WITH OTHER TESTS

The test results of this study are compared with those of Silver and Seed (1971) who tested a dry angular quartz sand (Crystal Silica No. 20). The comparison is made at a relative density of 60 percent and is shown in Figure 8a, b, and c for cycles, 1, 10, and 300, respectively. In general, the results show a similar trend but are lower than the data of Silver and Seed (1971). Although the data by Silver and Seed are for a D/H ratio of 4, the sample size is much smaller and may be the reason (besides angularity) for the difference in shear modulus at a given shear strain. This observation is consistent with the data presented in Figure 4 for clays and consistent with the observations of Carroll (1979).

Finally, the test results are compared with those of Silver and Seed with respect to damping in Figure 9a, b, and c for D/H ratios of 12, 6, and 3, respectively. Similar trends are observed.

DISCUSSION

Table 2 summarizes the effects of the D/H ratio on moduli and damping. For failure conditions (high shear strains), the D/H ratio does not affect the shear modulus; it only affects the hysteretic damping. The damping ratio is lower for a given shear strain at high D/H ratio.

The largest influence of the diameter to height ratio occurs at low shear strains (< 1 percent). As Casagrande stated, as cited by Seed (1979), "if test samples that are relatively thick and of small diameter are subjected to simple shear test conditions without the ability to develop complementary shear stresses on vertical boundaries ..., stress concentrations and nonuniformities in density distribution in the sample will lead to significant errors in test data." At low shear strains, the thick sample (D/H = 3) exhibits higher shear modulus than samples with a larger D/H ratio. Perhaps the higher shear modulus is caused by larger confining pressures which result from the induced normal forces on the top and bottom shear plates from the counter-balancing moments required for equilibrium as the specimen is sheared.

Using a higher shear modulus in earthquake response analysis could lead to predicted lower shear strains than what might actually occur. If one were to use the threshold shear strain approach for liquefaction potential (Dobry et al., 1980), an incorrect prediction on the unsafe side would result. Using an incorrect soil stiffness may result in incorrect pore pressure development prediction and the rate at which the pore pressures develop. Thus a higher than actual shear modulus determined from laboratory tests in which a low D/H ratio was used may provide predicted behavior which is on the unconservative side.

CONCLUSIONS

The following conclusions were drawn from the results of this test program.

(1) Sample size and D/H ratio affect the shear modulus and hysteretic damping ratio relationship with respect to shear strain.

(2) The above mentioned affect on the shear modulus is limited to low shear strains, less than 1 percent.

(3) At high shear strains (greater than 1 percent), the shear modulus appears unaffected by the D/H ratio. The values of shear modulus described herein are found to be lower than typical values reported in the literature and are not intended for use in design.

(4) In the shear strain range tested (from about 0.01 to 1 percent), the percent of critical hysteretic damping decreases for a given shear strain as cycle number and D/H ratio increase. The values of damping described herein are found to be higher than reported in the literature and are not intended for use in design.

Based upon these conclusions, the utilization of cyclic simple shear tests in which low D/H ratios (< 4) are used may result in predictions of ground response analysis and pore pressure development in saturated sands on the unsafe side. Further testing using a variety of soils, specimen preparation methods and devices with different D/H ratios should be undertaken to further corroborate the above mentioned findings. If a minimum D/H ratio of 6 is to be used as proposed by Kovacs (1973), then problems arise with obtaining undisturbed samples for an adequate sized simple shear device.

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Table 2. Effect of D/H Ratio in Moduli and Damping for Low and High Shear Strains

	Effect of D/H Ratio on	
Shear Strain	Moduli	Damping
Low (< 1 percent)	G larger with small samples (< 4 in) and larger with low D/H ratios (< 4).	λ_H larger with low D/H ratios (< 4).
High (< 1 percent)	G about same for range of D/H ratios studied.	Same as above.

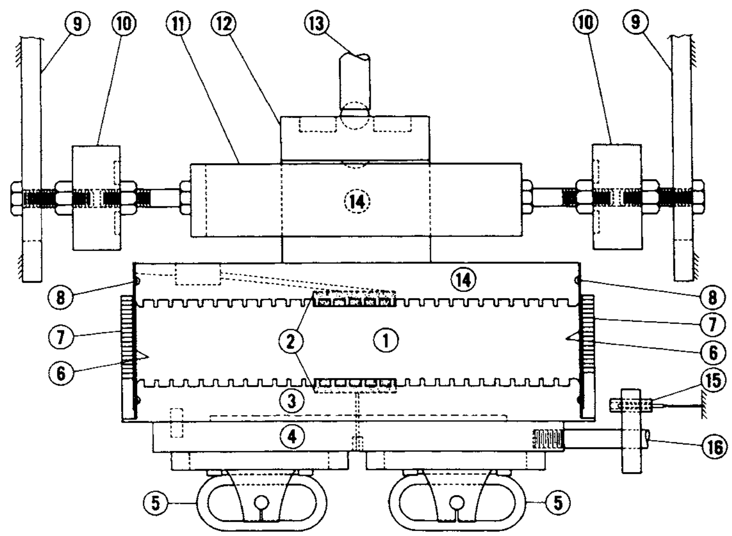


Fig. 1. Schematic diagram of 12 in diameter variable height cyclic simple shear device.

1 Sample (2 in. high shown) 2 Porous stone 3 Bottom loading plate with fins 4 Lower plate with shear key 5 Track-mounted Thompson bushings 6 Rubber membrane 7 Stacked rings (0.05 or 0.12 in. thick) 8 O ring seal 9 Reaction plate (attached to chamber, not shown; lock nut not shown) 10 Pair of 2 kip load cells with compression preload to measure horizontal force; average load used 11 Top (rectangular) reaction plate 12 12 kip load cell to measure normal force 13 Vertical pneumatic piston for applying normal force 14 Top loading plate with fins 15 LVDT for measurement of horizontal deformation 16 Horizontal piston to electrohydraulic actuator with load cell to measure horizontal force (not shown).

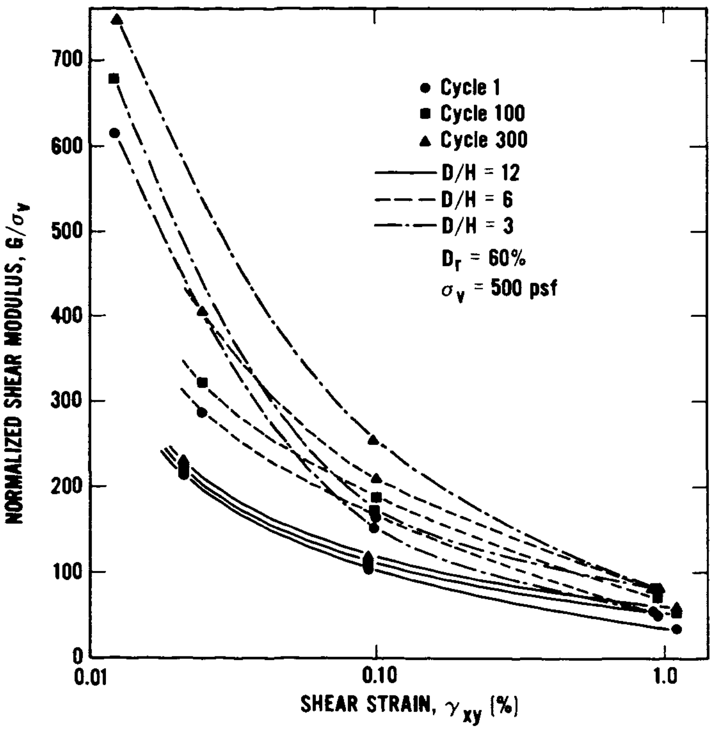


Fig. 2. Normalized shear modulus versus shear strain for sand at three D/H ratios.

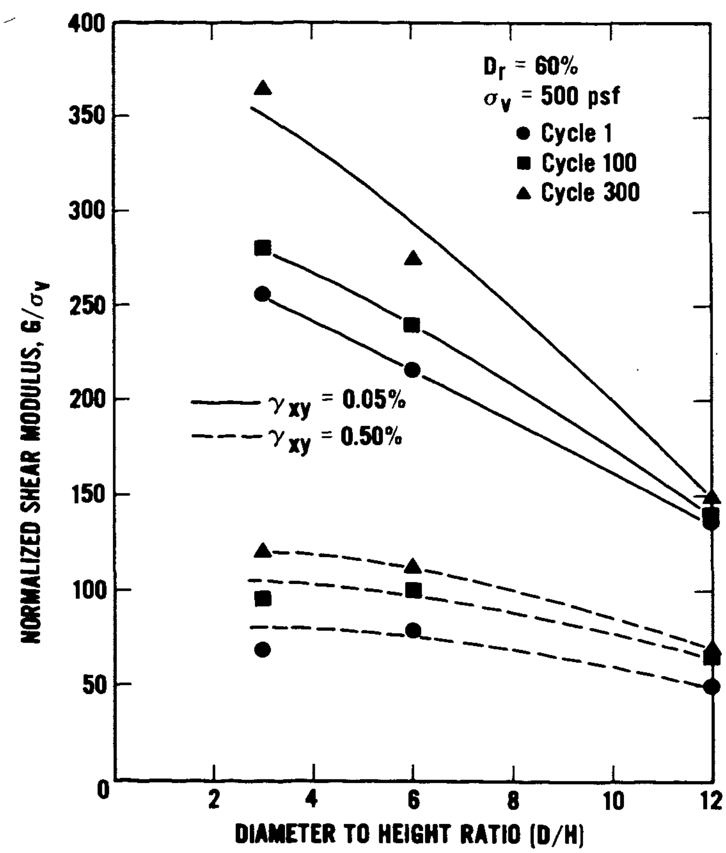


Fig. 3. Normalized shear modulus versus D/H ratio for sand.

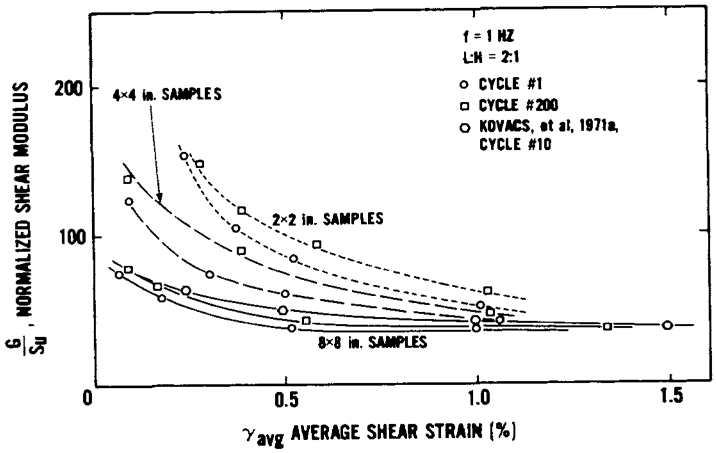


Fig. 4. Normalized shear modulus versus average shear strain for unconfined blocks of kaolinite/bentonite mix (after Kovacs, 1973).

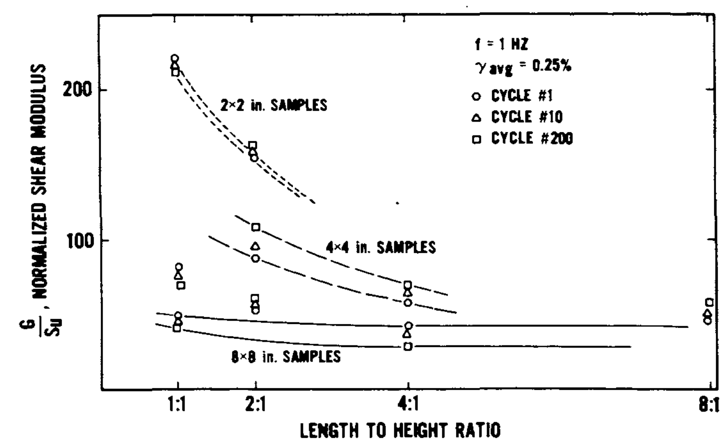


Fig. 5. Normalized shear modulus versus length to height ratio for unconfined blocks of kaolinite/bentonite mix for an average shear strain of 0.25 percent (after Kovacs, 1973).

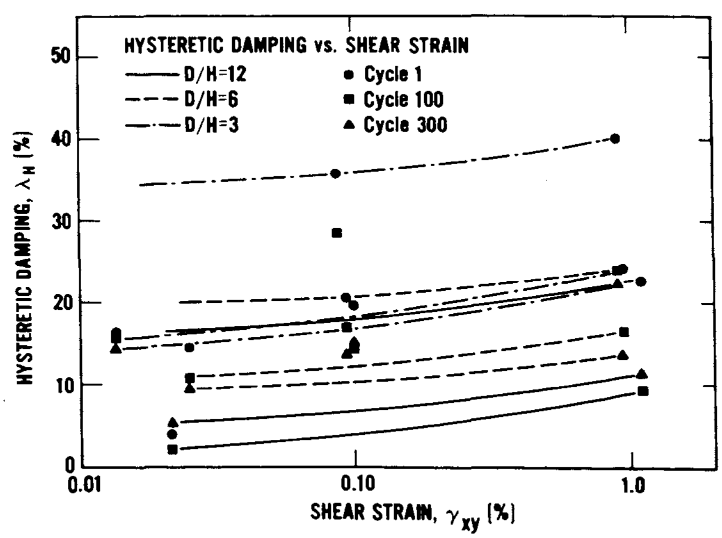


Fig. 6. Percent of critical hysteretic damping versus shear strain for sand at three D/H ratios.

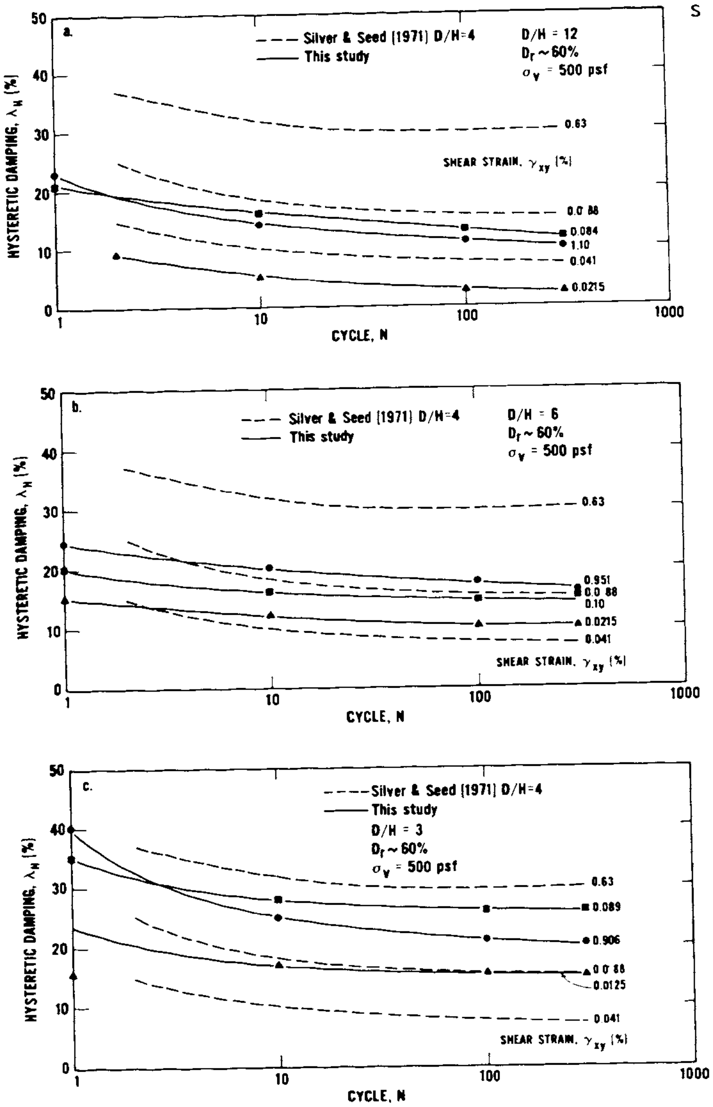
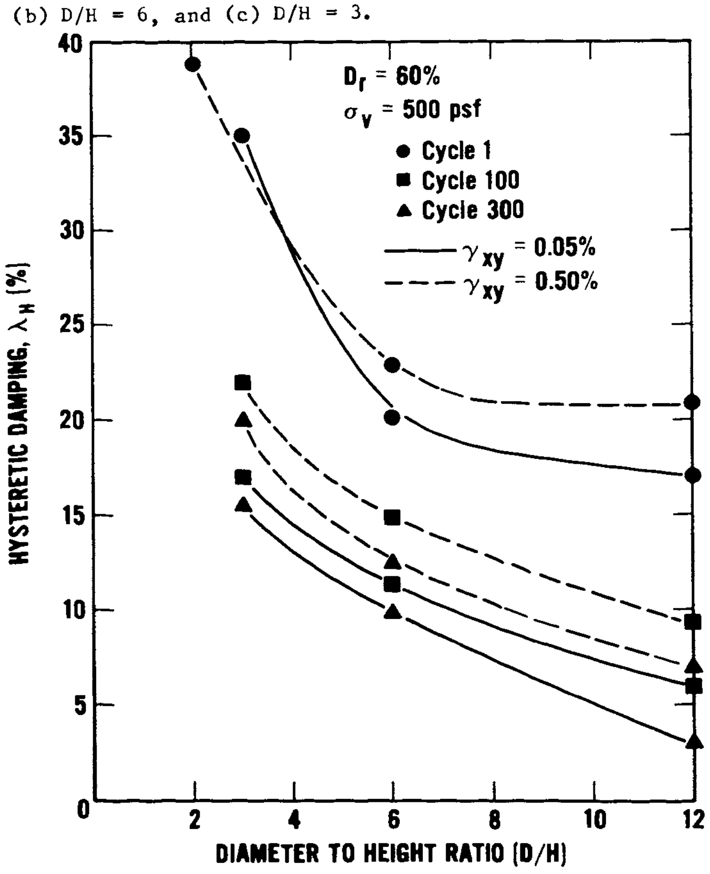


Fig. 9. Comparison of hysteretic damping versus cycle number for sand at several D/H ratios. (a) D/H = 12, (b) D/H = 6, and (c) D/H = 3.



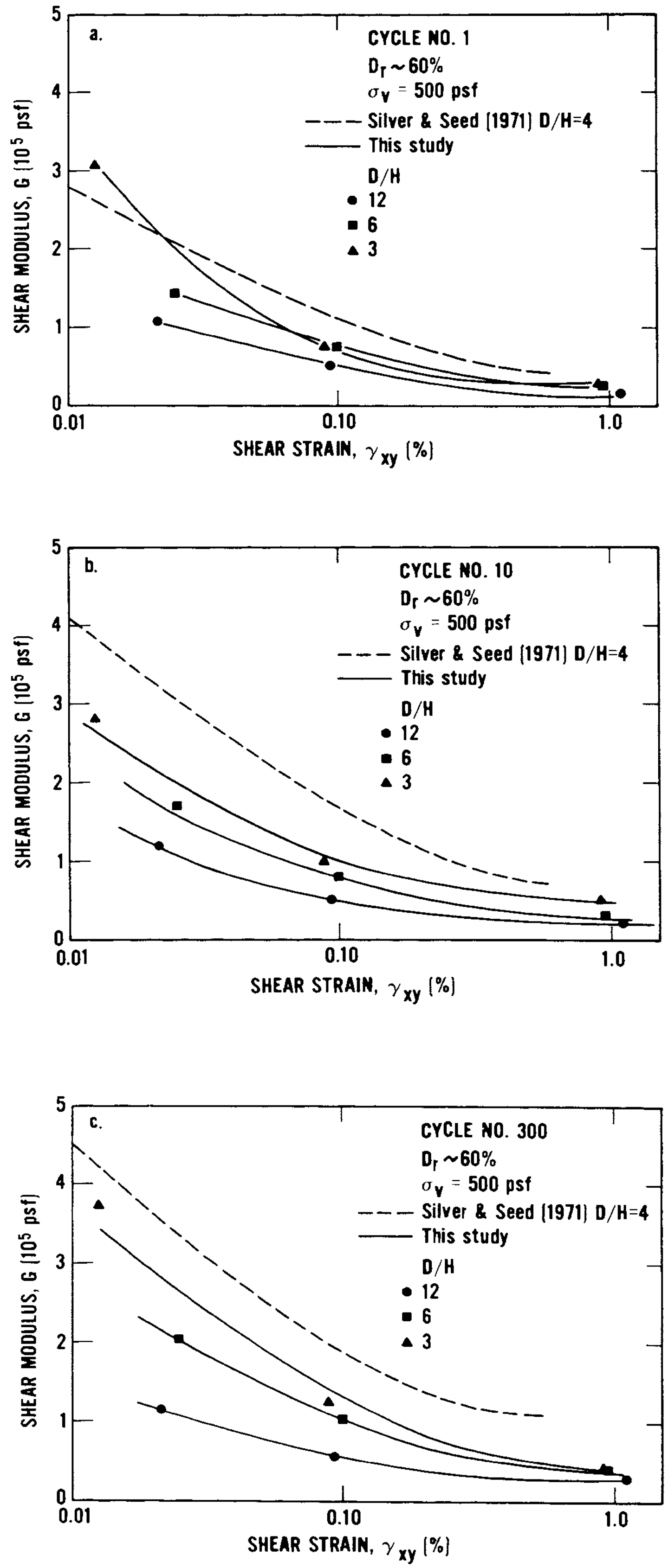


Fig. 8. Comparison of shear modulus versus shear strain for sand at several D/H ratios. (a) cycle 1, (b) cycle 10, and (c) cycle 300.